STRUCTURAL CALCULATIONS

FOR

LOFT CONVERSION WITH REAR DORMER

AT

SITE POST CODE: SE19 3HX

ON

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BY

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REFERENCE:

CALCULATIONS ARE PREPARED IN ACCORDANCE WITH THE FOLLOWING CODES OF PRACTICE:

BS 5268 [TIMBER SC3 GRADE]
CODE OF PRACTICE FOR USE OF TIMBER

BS 648 [WEIGHT OF BUILDING MATERIAL]
CODE OF PRACTICE FOR BUILDING MATERIAL

BS 5950 PART 1 – 2000
STRUCTURAL USE OF STEELWORK IN BUILDING

BS 5628 PART 1 – 1992
CODE OF PRACTICE FOR USE OF MASONRY

BS 6399 PART 1 – 1996
LOADING FOR BUILDINGS
[DEAD AND IMPOSED LOADS]

BS 6399 PART 3 - 1988 LOADING FOR BUILDINGS
[IMPOSED ROOF LOAD]

SPAN TABLES FOR SOLID TIMBER MEMBERS IN FLOORS, CEILINGS AND ROOFS FOR DWELLINGS --TRADA 2004

LOAD CALCULATIONS:

FLAT ROOF

	DEAD LOAD (kN/m^2)	**LIVE LOAD (kN/m^2)**
PLASTER BOARD	0.12	SNOW & REPAIR ACCESS = 0.75 kN/m^2
JOIST	0.18	-
FELT	0.15	-
CHIPPINGS	0.24	-
BOARDING	0.12	-
TOTAL	**0.81 kN/m^2**	**0.75 kN/m^2**

PITCHED ROOF

	DEAD LOAD (kN/m^2)	**LIVE LOAD (kN/m^2)**
RAFTERS	0.12	SNOW & REPAIR ACCESS = 0.75 kN /m2
BATTENS	0.04	-
FELT	0.05	-
TILES	0.68	-
	0.89 (On Slope)	-
	0.89 / Cos 45 Degrees	-
TOTAL	**= 1.26 (On Plan)**	**0.75 kN/m^2**

FLOOR

	DEAD LOAD (kN/m^2)	**LIVE LOAD (kN/m^2)**
CEILING	0.15	IMPOSED = 1.50 kN /m2
JOIST	0.15	-
BOARDING	0.20	-
TOTAL	**0.50 kN/m^2**	**1.50 kN/m^2**

CEILING

	DEAD LOAD (kN/m^2)	LIVE LOAD (kN/m^2)
CEILING	0.25	-
TOTAL	**0.25 kN/m^2**	-

BRICK WORK

	DEAD LOAD (kN/m^3)	LIVE LOAD (kN/m^3)
BRICKS	22.00	-
TOTAL	**22.00 kN**/m^3	-

BLOCK WORK

	DEAD LOAD (kN/m^3)	LIVE LOAD (kN/m^3)
BLOCKS	14.00	-
TOTAL	**14.00 kN**/m^3	-

INTERNAL STUDDING

	DEAD LOAD (kN/m^2)	LIVE LOAD (kN/m^2)
PLASTER BOARD	0.04	-
STUD	0.68	-
TOTAL	**0.72 kN/m^2**	-

EXTERNAL STUDDING

	DEAD LOAD (kN/m^2)	**LIVE LOAD (kN/m^2)**
PLASTER BOARD	0.12	-
TILING	0.68	-
FELT	0.05	-
BATTENS	0.04	-
STUD	0.12	-
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TOTAL	**1.01 kN/m^2**	-
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GENERAL AND SAFETY NOTES:

1. All dimensions, setting out and levels are to be verified on site with the architect prior to the commencement of any site work.

2. Building control approval must be obtained prior to the commencement of building works.

3.The contractor shall be responsible for and must take all necessary precautions to ensure the stability of the existing structure and earthworks on adjoining sites during the course of the contract.

4.Materials and constructions are to be in accordance with the relevant British Standards and Codes of Practice.

5.Any services or drainage which pass through the foundation are to be encased in a flexible sleeve.

6.The dimensions of all steel sections required should be measured on site by the client (or their representative contractor or steelwork fabricator). All Steel Beams to have minimum of 1/2hr Fire Resistance via 'Nullifire' Paint or 19mm Gyproc Plank tied with 1.6mm wire binding@100mm c/c and finished in Carlite Bonding 16mm thick.

7. It is the responsibility of the Client / Contractor to notify the designer of any discrepancies.

8. Note that all steel (Sizes & Quantity) shown on the drawings are should be checked prior to ordering any material.

9. Structural Drawings provided here are based on architectural drawings and site survey therefore are indicative only and are NOT TO SCALE.

10. All existing foundations and lintels to be exposed and to be checked for adequacy by Builder / Building Control Officer and or replaced if necessary.

11. Depth of all footings is to be approved by the local building inspector. Minimum of 1 m deep, advise engineer if footings are located within 30m of large trees or hedges. Also notify if water is struck, tree roots are found or clay soil appears very dry and brittle.

LOAD CALCULATIONS FOR ROOF BEAM (RB):

CLEAR SPAN OF THE ROOF BEAM (RB) = 5450mm

LOADING:

DEAD LOAD ON ROOF BEAM (RB):

PITCHED ROOF = 1.26 kN/m^2 x SPAN OF ROOF IN MTS.

= 1.26 kN/m^2 x 1.5m

= 1.89 kN/m

FLAT ROOF = 0.81 kN/m x SPAN OF ROOF IN MTS.

= 0.81 kN/m^2 x 1.8m

= 1.46 kN/m

THEREFORE TOTAL DEAD LOAD ON RB = 1.89 + 1.46 = 3.35 kN/m.

LIVE LOAD ON ROOF BEAM (RB):

PITCHED ROOF = 0.75 kN/m^2 x SPAN OF ROOF IN MTS.

= 0.75 kN/m^2 x 1.5m

= 1.13 kN/m

FLAT ROOF = 0.75 kN/m^2 x SPAN OF ROOF IN MTS.

= 0.75 kN/m^2 x 1.8m

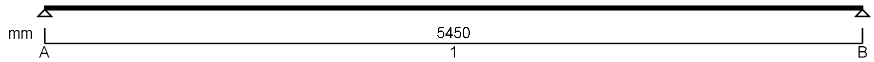
= 1.35 kN/m

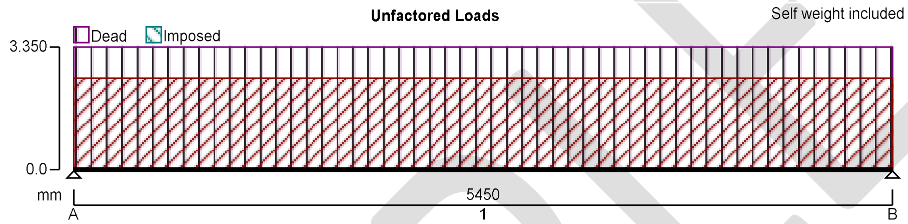
THEREFORE TOTAL LIVE LOAD ON RB = 1.13 + 1.35 = 2.48 kN/m

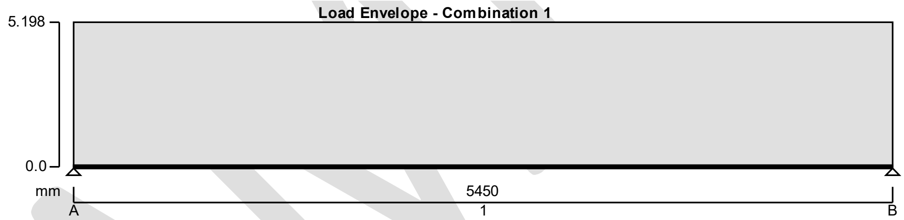
STRUCTURAL REPORT FOR ROOF BEAM (RB):

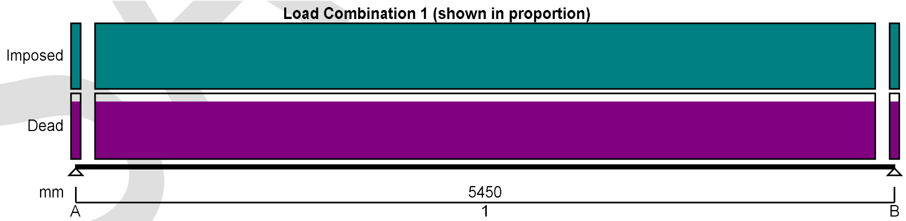
STEEL BEAM ANALYSIS & DESIGN (BS5950)

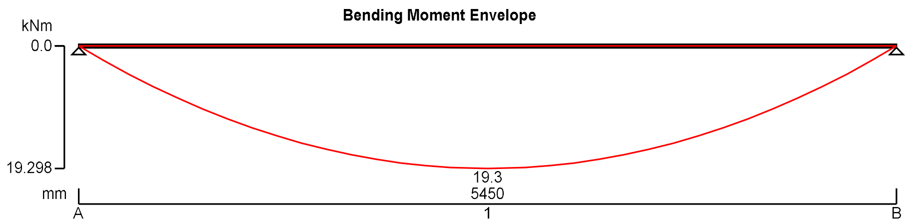
TEDDS calculation version 1.0.05

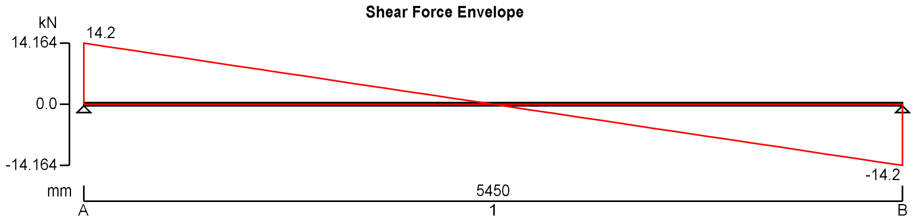












Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads

Dead self weight of beam × 1

Dead full UDL 3.35 kN/m

Live full UDL 2.5 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
	Span 1	Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

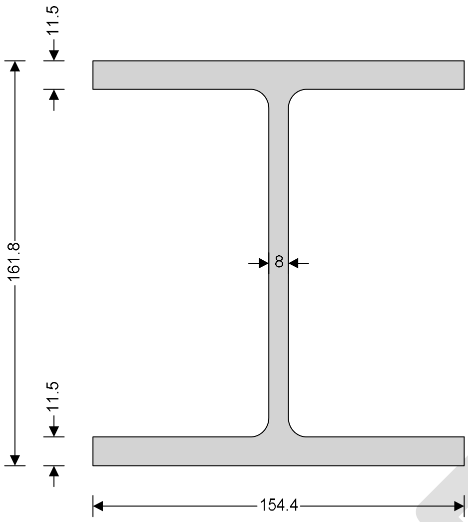
Maximum moment	M_{max} = **19.3** kNm	M_{min} = **0** kNm
Maximum moment span1	M_{s1_max} = **19.3** kNm	M_{s1_min} = **0** kNm
Maximum shear	V_{max} = **14.2** kN	V_{min} = **-14.2** kN
Maximum shear span1	V_{s1_max} = **14.2** kN	V_{s1_min} = **-14.2** kN
Deflection span1	δ_{s1_max} = **9.4** mm	δ_{s1_min} = $\mathbf{2.7\times10^{-15}}$ mm
Reactions at support A	R_{A_max} = **14.2** kN	R_{A_min} = **14.2** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **10.1** kN	
Reactions at support B	R_{B_max} = **14.2** kN	R_{B_min} = **14.2** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **10.1** kN	

Section details

Section type	**UC 152x152x37**
Steel grade	**S355**

From table 9: Design strength p_y

Thickness of element	max(T, t) = **11.5** mm
Design strength	p_y = **355** N/mm^2
Modulus of elasticity	E = **205000** N/mm^2



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis K_x = **1.00**

Effective length factor in minor axis K_y = **1.00**

Effective length factor for lateral-torsional buckling $K_{LT.A}$ = **1.00**

$K_{LT.B}$ = **1.00**

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275\ N/mm^2 / p_y]}$ = **0.88**

Internal compression parts - Table 11

Depth of section d = **123.6** mm

$d / t = 17.6 \times \varepsilon <= 80 \times \varepsilon$ Class 1 plastic

Outstand flanges - Table 11

Width of section b = B / 2 = **77.2** mm

$b / T = 7.6 \times \varepsilon <= 9 \times \varepsilon$ Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force $F_v = max(abs(V_{max}), abs(V_{min}))$ = **14.2** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area $A_v = t \times D$ = **1294** mm^2

Design shear resistance $P_v = 0.6 \times p_y \times A_v$ = **275.7** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment $M = max(abs(M_{s1\_max}), abs(M_{s1\_min}))$ = **19.3** kNm

Moment capacity - Section 4.2.5

Moment capacity low shear - cl.4.2.5.2 $M_c = min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx})$ = **109.6** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling $L_E = 1.0 \times L_{s1}$ = **5450** mm

Slenderness ratio $\lambda = L_E / r_{yy}$ = **140.761**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter u = **0.848**

Torsional index x = **13.334**

Slenderness factor $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = \mathbf{0.625}$

Ratio - cl.4.3.6.9 $\beta_W = \mathbf{1.000}$

Equivalent slenderness - cl.4.3.6.7 $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} = \mathbf{74.567}$

Limiting slenderness - Annex B.2.2 $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = \mathbf{30.198}$

$\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant $\alpha_{LT} = \mathbf{7.0}$

Perry factor $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = \mathbf{0.311}$

Euler stress $p_E = \pi^2 \times E / \lambda_{LT}^2 = \mathbf{363.9}$ N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = \mathbf{415.9}$ N/mm^2

Bending strength - Annex B.2.1 $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = \mathbf{206.6}$ N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment $M_2 = \mathbf{14.5}$ kNm

Moment at centre-line of segment $M_3 = \mathbf{19.3}$ kNm

Moment at three quarter point of segment $M_4 = \mathbf{14.5}$ kNm

Maximum moment in segment $M_{abs} = \mathbf{19.3}$ kNm

Equivalent uniform moment factor for lateral-torsional buckling

$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = \mathbf{0.925}$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment $M_b = p_b \times S_{xx} = \mathbf{63.8}$ kNm

$M_b / m_{LT} = \mathbf{69}$ kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead, imposed and live loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 = \mathbf{15.1}$ mm

Maximum deflection span 1 $\delta = \max(abs(\delta_{max}), abs(\delta_{min})) = \mathbf{9.412}$ mm

PASS - Maximum deflection does not exceed deflection limit

LOAD CALCULATIONS FOR STAIR TRIMMER 1 & 2:

CLEAR SPAN OF STAIR TIMMER 1 & 2 = 1200mm

LOADING:

DEAD LOAD ON STAIR TRIMMER 1 & 2:

TIMBER FLOOR = 0.50 kN/m^2 x SPAN OF FLOOR IN MTS.

= 0.50 kN/m^2 x 2.00m

= 1.00 kN/m

STUD WORK = 0.50 kN /m^2 x HEIGHT OF STUD PARTITION IN MTS.
(100mm)

= 0.50 kN /m^2 x 2.3m

= 1.15kN/m

THEREFORE TOTAL DEAD LOAD ON STAIR TRIMMER 1 = 1.00 +1.15 =2.15 kN/m

LIVE LOAD ON STAIR TRIMMER 1 & 2:

TIMBER FLOOR = 1.5 kN/m^2 x SPAN OF FLOOR IN MTS.

= 1.5 kN/m^2 x 2.00m

= 3.00 kN/m

THEREFORE TOTAL LIVE LOAD ON TRIMMER 1= 3.00 kN/m

STRUCTURAL REPORT FOR STAIR TRIMMER 1 & 2:

Analysis for a simply-supported single-span timber beam to BS 5268

TEDDS calculation version 1.0.02

Span length & partial factors for loading

Span (mm)	Factors for moments & forces			Factors for deflection		
	γ_{fd}	γ_{fi}	γ_{fw}	γ_{dd}	γ_{di}	γ_{dw}
1200	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**

Loading data (unfactored)

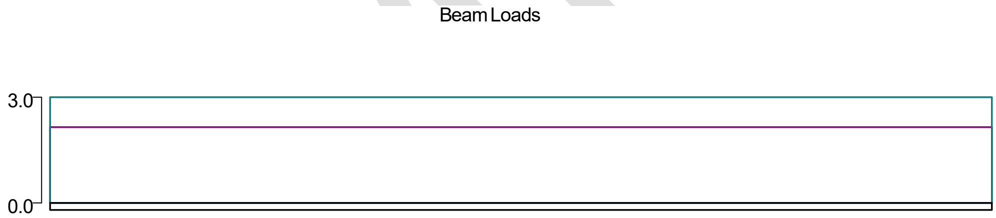
Ref.	Category	Type	Load kN/m	Position mm	Load kN/m	Position mm
1	**"Dead"**	UDL	**2.2**	**0**	-	**1200**
2	**"Imposed"**	UDL	**3.0**	**0**	-	**1200**

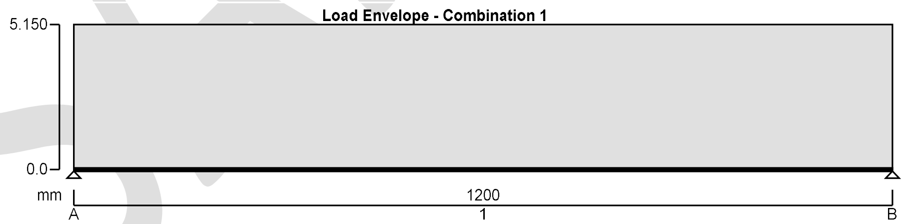
Analysis results - entire span

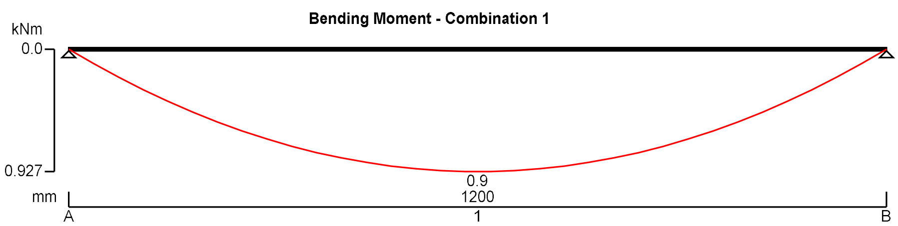
R_a	R_b	V	M		Deflection: δEI	
kN (fac)	kN (fac)	kN (fac)	kNm (fac)	Sense	kNm^3	Direction
3.1	**3.1**	**3.1**	**0.9**	**"Sagging"**	**0.14**	**"Down"**

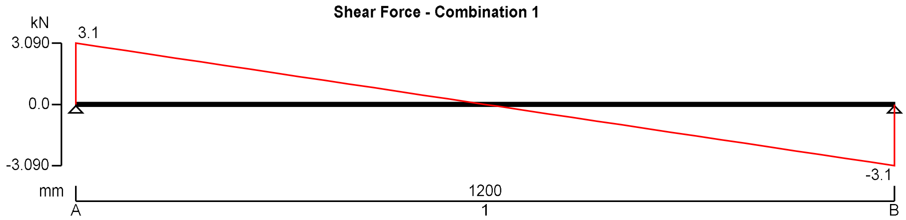
Unfactored support reactions

Support A	Dead load **-1.3** kN	Live load **-1.8** kN	Wind load **0.0** kN
Support B	Dead load **-1.3** kN	Live load **-1.8** kN	Wind load **0.0** kN









Member design checks for a simply-supported single-span timber beam to BS 5268

Timber member design BS 5268-2:2002

Summary of results				
Section size	D = **150** mm	B = **100** mm	A= **15000** mm^2	
Section properties (x-x)	I_{xx} = **28125000** mm^4	Z_{xx} = **375000** mm^3	r_{xx} = **43.3** mm	
(y-y)	I_{yy} = **12500000** mm^4	Z_{yy} = **250000** mm^3	r_{yy} = **28.9** mm	
Grade	**"C24"**	σ_c = **7.90** N/ mm^2		
Check	*Stress*	*Capacity*	*Notes*	*Result*
Bending stress	$\sigma_{m.a.para}$ = M / Z_{xx} = **2.47** N/mm^2	$\sigma_{m.adm.para}$ = **8.09** N/mm^2	Moment M = **0.9** kNm	**Pass**
Shear stress	τ_a = **0.31** N/mm^2	τ_{adm} = **0.71** N/mm^2	Shear V = **3.1** kN	**Pass**
Deflection	δ = **0.6** mm	14 mm		**Pass**
	δ / L_s = **0.00047**	0.003		**Pass**

LOAD CALCULATIONS FOR STAIR TRIMMER 3:

CLEAR SPAN OF STAIR TIMMER 3 = 4800mm

LOADING:

DEAD LOAD ON STAIR TRIMMER 3:

TIMBER FLOOR = 0.50 kN/m^2 x SPAN OF FLOOR IN MTS.

= 0.50 kN/m^2 x 0.2m

= 0.10 kN/m

STUD WORK (100mm) = 0.5 kN/m^2 x HEIGHT OF STUD PARTITION IN MTS.

= 0.5 kN/m^2 x 2.3m

= 1.15 kN/m

THEREFORE TOTAL DEAD LOAD ON STAIR TRIMMER 3=0.10+1.15 = 1.25 kN/m

LIVE LOAD ON STAIR TRIMMER 3:

TIMBER FLOOR = 1.5 kN / m2 x SPAN OF FLOOR IN MTS.

= 1.5 kN/m^2 x 0.2m

= 0.30 kN/m

THEREFORE TOTAL LIVE LOAD ON STAIR TRIMMER 3 = 0.30 kN/m

POINT LOAD FROM STAIR TRIMMER 1 = 3.10 kN@1.40from L.H.S

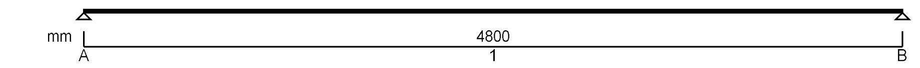
POINT LOAD FROM STAIR TRIMMER 2 = 3.10 kN@3.65from L.H.S

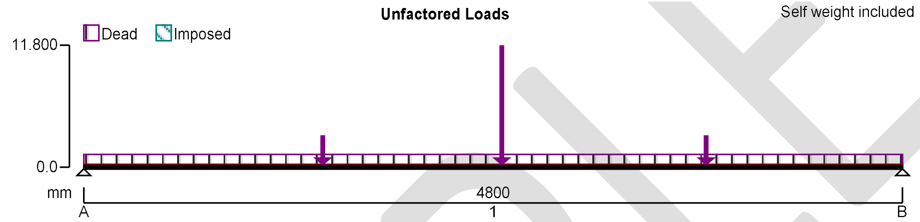
POINT LOAD FROM LOFT BEAM, LB1 = 11.80 kN@2.45from L.H.S

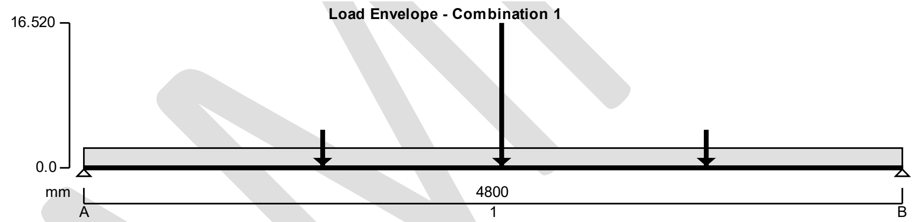
STRUCTURAL REPORT FOR STAIR TRIMMER 3:

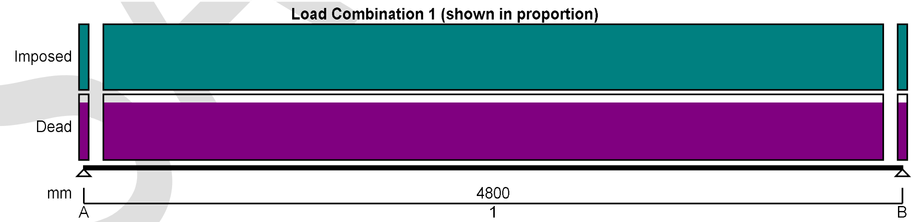
STEEL BEAM ANALYSIS & DESIGN (BS5950)

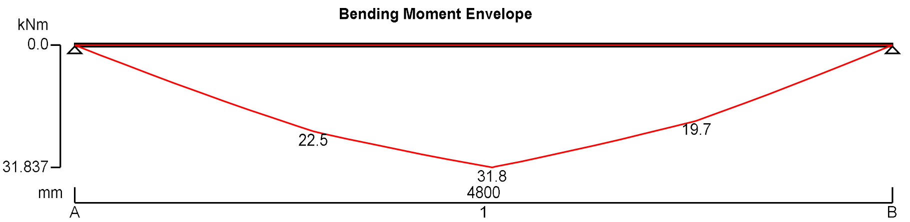
TEDDS calculation version 1.0.05

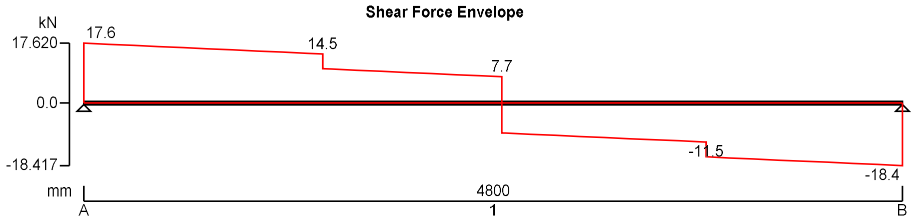












Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 1.25 kN/m
Live full UDL 0.3 kN/m
Dead point load 3.1 kN at 1400 mm
Dead point load 3.1 kN at 3650 mm
Dead point load 11.8 kN at 2450 mm

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
	Span 1	Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

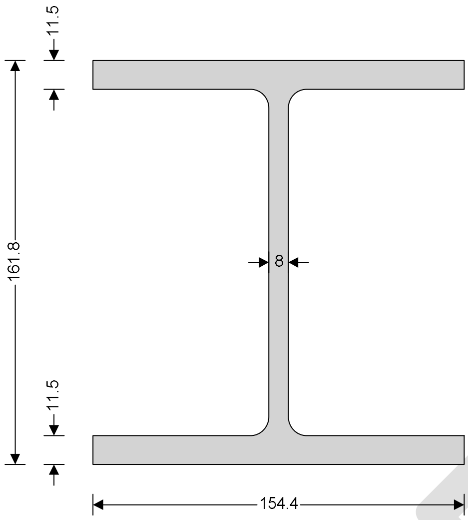
Maximum moment	M_{max} = **31.8** kNm	M_{min} = **0** kNm
Maximum moment span1	M_{s1_max} = **31.8** kNm	M_{s1_min} = **0** kNm
Maximum shear	V_{max} = **17.6** kN	V_{min} = **-18.4** kN
Maximum shear span1	V_{s1_max} = **17.6** kN	V_{s1_min} = **-18.4** kN
Deflection span1	δ_{s1_max} = **10.7** mm	δ_{s1_min} = **3.7×10^{-15}** mm
Reactions at support A	R_{A_max} = **17.6** kN	R_{A_min} = **17.6** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **12.6** kN	
Reactions at support B	R_{B_max} = **18.4** kN	R_{B_min} = **18.4** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **13.2** kN	

Section details

Section type	**UC 152x152x37**
Steel grade	**S355**

From table 9: Design strength p_y

Thickness of element	max(T, t) = **11.5** mm
Design strength	p_y = **355** N/mm^2
Modulus of elasticity	E = **205000** N/mm^2



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis — K_x = **1.00**

Effective length factor in minor axis — K_y = **1.00**

Effective length factor for lateral-torsional buckling — $K_{LT.A}$ = **1.00**

$K_{LT.B}$ = **1.00**

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **0.88**

Internal compression parts - Table 11

Depth of section — d = **123.6** mm

$d / t = 17.6 \times \varepsilon <= 80 \times \varepsilon$ — Class 1 plastic

Outstand flanges - Table 11

Width of section — b = B / 2 = **77.2** mm

$b / T = 7.6 \times \varepsilon <= 9 \times \varepsilon$ — Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force — $F_v = max(abs(V_{max}), abs(V_{min}))$ = **18.4** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area — $A_v = t \times D$ = **1294** mm^2

Design shear resistance — $P_v = 0.6 \times p_y \times A_v$ = **275.7** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment — $M = max(abs(M_{s1\_max}), abs(M_{s1\_min}))$ = **31.8** kNm

Moment capacity - Section 4.2.5

Moment capacity low shear - cl.4.2.5.2 — $M_c = min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx})$ = **109.6** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling — $L_E = 1.0 \times L_{s1}$ = **4800** mm

Slenderness ratio — $\lambda = L_E / r_{yy}$ = **123.973**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter — u = **0.848**

Torsional index — x = **13.334**

Slenderness factor $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25}$ = **0.658**

Ratio - cl.4.3.6.9 β_W = **1.000**

Equivalent slenderness - cl.4.3.6.7 $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]}$ = **69.230**

Limiting slenderness - Annex B.2.2 $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5}$ = **30.198**

λ_{LT} > λ_{L0} - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant α_{LT} = **7.0**

Perry factor $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0)$ = **0.273**

Euler stress $p_E = \pi^2 \times E / \lambda_{LT}^2$ = **422.1** N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2$ = **446.2** N/mm^2

Bending strength - Annex B.2.1 $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5})$ = **224.3** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment M_2 = **19.5** kNm

Moment at centre-line of segment M_3 = **31.4** kNm

Moment at three quarter point of segment M_4 = **20.3** kNm

Maximum moment in segment M_{abs} = **31.8** kNm

Equivalent uniform moment factor for lateral-torsional buckling

$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44)$ = **0.881**

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment $M_b = p_b \times S_{xx}$ = **69.2** kNm

M_b / m_{LT} = **78.6** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead, imposed and live loads

Limiting deflection $\delta_{lim} = L_{s1} / 360$ = **13.3** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **10.725** mm

PASS - Maximum deflection does not exceed deflection limit

LOAD CALCULATIONS FOR 100 mm STEEL POST (SHS):

Try 100x100x5mm Steel Posts (SHS)

Length of Steel Post = 2600mm

Grade of Steel = "S355"

Load from Roof Beam = 25.00 kN

STRUCTURAL REPORT FOR 100mm STEEL POST (SHS):

STEEL MEMBER DESIGN TO BS5950: PART 1: 2000

Member design checks for a steel member to BS 5950:2000

Section properties

Try SHS 100x100x5.0 section.

D = **100** mm B = **100** mm $T = t$ = **5.0** mm $A_g = A$ = **18.7** cm^2

$d_x = D - 3 \times t$ = **85.0** mm $b_x = B - 3 \times t$ = **85.0** mm $b_y = d_x$ $d_y = b_x$

$A_{vy} = A \times D / (D + B)$ = **9.4** cm^2 $A_{vx} = A \times B / (D + B)$ = **9.4** cm^2

I_x = **279** cm^4 I_y = **279** cm^4 r_x = **3.86** cm r_y = **3.86** cm

S_x = **66.4** cm^3 S_y = **66.4** cm^3 Z_x = **55.9** cm^3 Z_y = **55.9** cm^3

$d_{vx} = A_{vx} / (2 \times t)$ = **93.7** mm $d_{vy} = A_{vy} / (2 \times t)$ = **93.7** mm

$S_{vx} = 2 \times t \times d_{vy}^2 / 4$ = **21.9** cm^3 $S_{vy} = 2 \times t \times d_{vx}^2 / 4$ = **21.9** cm^3

Strut curve (a) for x-axis and y-axis. (Tables 23 & 24) $z_1 = 5/3$ $z_2 = 5/3$

Steel grade **"S355"** p_y = **355** N/mm^2 $p_{yw} = p_y$ $\varepsilon = \sqrt{(275 N/mm^2 / p_y)}$ = **0.880** K_e = **1.1**

Cl. 3.1.1 & 3.4.3

Geometry

Member has no intermediate restraint.

L = **2600** mm

Buckling:	Restraint:	End 1:	End 2:
Normal to x-axis	Position	**X**	**X**
	Direction		
Normal to y-axis	Position	**X**	**X**
	Direction		

For strut buckling normal to x-axis K_x = **1.00** L_{Ex} = **2600** mm

For strut buckling normal to y-axis K_y = **1.00** L_{Ey} = **2600** mm

Cl. 4.3.5 & 4.7.3

Loading

Internal forces & moments on member under factored loading for uls design:

F_t = 0 kN F_c = **25.00** kN $n = F_c / (A \times p_y)$ = **0.038**

F_{vx} = 0 kN

F_{vy} = 0 kN

M_x = 0 kNm

M_y = 0 kNm

Section classification

b_x / t = **17.0** d_x / t = **17.0** b_y / t = **17.0** d_y / t = **17.0**

$r_{1x} = \min(1.0, \max(-1.0, F_c / (2 \times d_x \times t \times p_{yw})))$ = **0.083**

$r_{1y} = \min(1.0, \max(-1.0, F_c / (2 \times d_y \times t \times p_{yw})))$ = **0.083**

$r_2 = F_c / (A_g \times p_{yw})$ = **0.038**

Section classification is Plastic

Cl. 3.5.1 - 3.5.5

Compression resistance - strut buckling about x-axis

F_c = **25.0** kN L_{Ex} = **2.60** m $\lambda_x = L_{Ex} / r_x$ = **67**

Strut curve (a) applies $\lambda_0 = 0.2 \times (\pi^2 \times E_{S5950} / p_y)^{0.5}$ = **15** a_x = **2.0**

$\eta_x = \max(0, a_x \times (\lambda_x - \lambda_0) / 1000)$ = **0.104** $p_{ex} = \pi^2 \times E_{S5950} / \lambda_x^2$ = **446** N/mm²

$\phi_x = (p_y + (\eta_x + 1) \times p_{ex}) / 2$ $p_{cx} = p_{ex} \times p_y / (\phi_x + (\phi_x^2 - p_{ex} \times p_y)^{0.5})$ = **278** N/mm²

p_{cx} = **278** N/mm² $P_{cx} = A \times p_{cx}$ = **520.8** kN

Check $F_c \leq P_{cx}$ Pass - Compression

Slenderness less than 180

Cl. 4.7.4 & Annex C

Compression resistance - strut buckling about y axis

F_c = **25.0** kN L_{Ey} = **2.60** m $\lambda_y = L_{Ey} / r_y$ = **67**

Strut curve (a) applies $\lambda_0 = 0.2 \times (\pi^2 \times E_{S5950} / p_y)^{0.5}$ = **15** a_y = **2.0**

$\eta_y = \max(0, a_y \times (\lambda_y - \lambda_0) / 1000)$ = **0.104** $p_{ey} = \pi^2 \times E_{S5950} / \lambda_y^2$ = **446** N/mm²

$\phi_y = (p_y + (\eta_y + 1) \times p_{ey}) / 2$ $p_{cy} = p_{ey} \times p_y / (\phi_y + (\phi_y^2 - p_{ey} \times p_y)^{0.5})$ = **278** N/mm²

$P_{cy} = A \times p_{cy}$ = **520.8** kN

Check $F_c \leq P_{cy}$ Pass - Compression

Slenderness less than 180

Cl. 4.7.4 & Annex C

Results summary

Summary of results

Material	Grade = **"S355"**	p_y = **355** N/mm²	
Section	**"SHS 100x100x5.0"**	Classification	**"Plastic"**

Check	*Load*	*Capacity*	*Notes*	*Result*
Strut buckling (x-axis)	F_c = **25.0** kN	P_{cx} = **520.8** kN	L_{Ex} = **2.6** m **Slenderness < 180**	**Pass**
Strut buckling (y-axis)	F_c = **25.0** kN	P_{cy} = **520.8** kN	L_{Ey} = **2.6** m **Slenderness < 180**	**Pass**

LOAD CALCULATIONS FOR LOFT BEAM (LB1):

CLEAR SPAN OF THE LOFT BEAM (LB1) = 4500mm

LOADING:

DEAD LOAD ON LOFT BEAM (LB1):

TIMBER FLOOR = 0.50 kN/m^2 x SPAN OF FLOOR IN MTS.

= 0.50 kN/m^2 x 2.5m

= 1.25 kN/m

STUD WORK = 0.5 kN/m^2 x HEIGHT OF STUD PARTITION IN MTS.
(100mm)

= 0.5 kN/m^2 x 2.3m

= 1.15kN/m

THEREFORE TOTAL DEAD LOAD ON LB1 = 1.25 + 1.15 = 2.40 kN/m.

LIVE LOAD ON LOFT BEAM, LB1:

TIMBER FLOOR = 1.50 kN/m^2 x SPAN OF FLOOR IN MTS.

= 1.5 kN/m^2 x 2.5m

= 3.75 kN /m

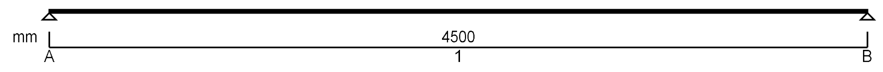
THEREFORE TOTAL LIVE LOAD ON LB1 = 3.75 kN/m

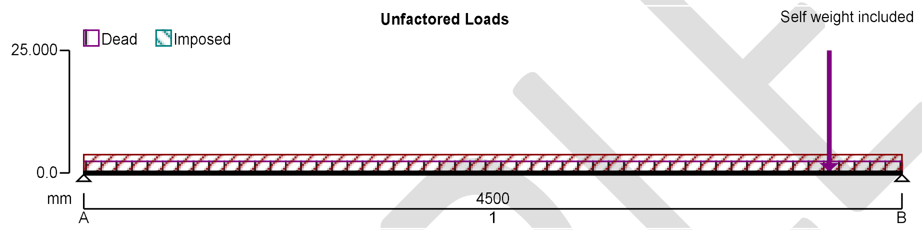
POINT LOAD FROM TIMBER POST = 25.00 kN@4.10m from L.H.S

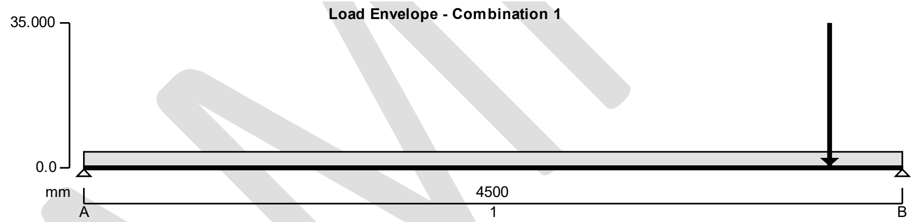
STRUCTURAL REPORT FOR LOFT BEAM (LB1):

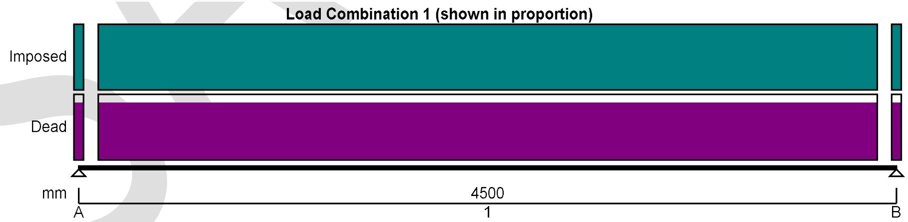
STEEL BEAM ANALYSIS & DESIGN (BS5950)

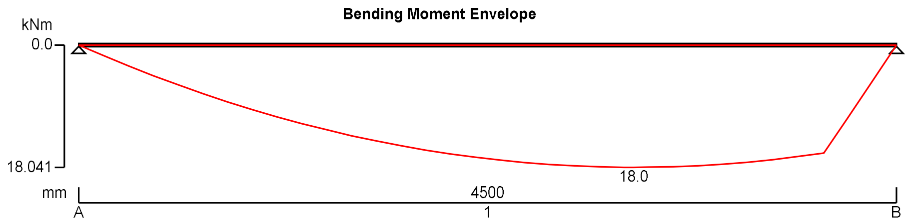
TEDDS calculation version 1.0.05

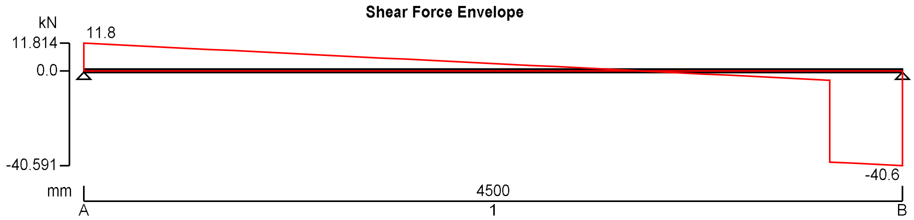












Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 2.4 kN/m
Live full UDL 3.75 kN/m
Dead point load 25 kN at 4100 mm

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
	Span 1	Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

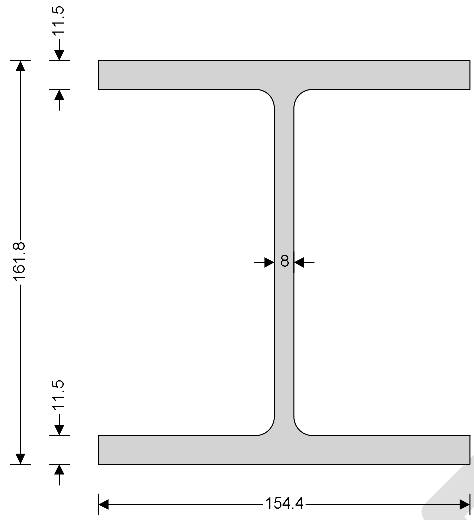
Maximum moment	M_{max} = **18** kNm	M_{min} = **0** kNm
Maximum moment span1	M_{s1_max} = **18** kNm	M_{s1_min} = **0** kNm
Maximum shear	V_{max} = **11.8** kN	V_{min} = **-40.6** kN
Maximum shear span1	V_{s1_max} = **11.8** kN	V_{s1_min} = **-40.6** kN
Deflection span1	δ_{s1_max} = **6** mm	δ_{s1_min} = **0** mm
Reactions at support A	R_{A_max} = **11.8** kN	R_{A_min} = **11.8** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **8.4** kN	
Reactions at support B	R_{B_max} = **40.6** kN	R_{B_min} = **40.6** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **29** kN	

Section details

Section type	**UC 152x152x37**
Steel grade	**S355**

From table 9: Design strength p_y

Thickness of element	max(T, t) = **11.5** mm
Design strength	p_y = **355** N/mm^2
Modulus of elasticity	E = **205000** N/mm^2



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis K_x = **1.00**

Effective length factor in minor axis K_y = **1.00**

Effective length factor for lateral-torsional buckling $K_{LT.A}$ = **1.00**

$K_{LT.B}$ = **1.00**

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **0.88**

Internal compression parts - Table 11

Depth of section d = **123.6** mm

$d / t = 17.6 \times \varepsilon <= 80 \times \varepsilon$ Class 1 plastic

Outstand flanges - Table 11

Width of section b = B / 2 = **77.2** mm

$b / T = 7.6 \times \varepsilon <= 9 \times \varepsilon$ Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **40.6** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area $A_v = t \times D$ = **1294** mm^2

Design shear resistance $P_v = 0.6 \times p_y \times A_v$ = **275.7** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **18** kNm

Moment capacity - Section 4.2.5

Moment capacity low shear - cl.4.2.5.2 $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx})$ = **109.6** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling $L_E = 1.0 \times L_{s1}$ = **4500** mm

Slenderness ratio $\lambda = L_E / r_{yy}$ = **116.225**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter u = **0.848**

Torsional index x = **13.334**

Slenderness factor $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25}$ = **0.676**

Ratio - cl.4.3.6.9 β_W = **1.000**

Equivalent slenderness - cl.4.3.6.7 $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]}$ = **66.605**

Limiting slenderness - Annex B.2.2 $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5}$ = **30.198**

$\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant α_{LT} = **7.0**

Perry factor $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0)$ = **0.255**

Euler stress $p_E = \pi^2 \times E / \lambda_{LT}^2$ = **456.1** N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2$ = **463.7** N/mm^2

Bending strength - Annex B.2.1 $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5})$ = **233.3** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment M_2 = **10.8** kNm

Moment at centre-line of segment M_3 = **16.8** kNm

Moment at three quarter point of segment M_4 = **17.8** kNm

Maximum moment in segment M_{abs} = **18** kNm

Equivalent uniform moment factor for lateral-torsional buckling

$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44)$ = **0.904**

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment $M_b = p_b \times S_{xx}$ = **72** kNm

M_b / m_{LT} = **79.7** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead, imposed, live and other loads

Limiting deflection $\delta_{lim} = L_{s1} / 360$ = **12.5** mm

Maximum deflection span 1 $\delta = \max(abs(\delta_{max}), abs(\delta_{min}))$ = **6.049** mm

PASS - Maximum deflection does not exceed deflection limit

LOAD CALCULATIONS FOR LOFT BEAM (LB2):

CLEAR SPAN OF THE LOFT BEAM (LB2) = 5850mm

LOADING:

DEAD LOAD ON LOFT BEAM (LB2):

PITCHED ROOF = 1.26 kN/m^2 x SPAN OF ROOF IN MTS.

= 1.26 kN/m^2 x 2.2m

= 2.77 kN/m

TIMBER FLOOR = 0.50 k kN/m^2 x SPAN OF FLOOR IN MTS.

= 0.50 kN/m^2 x 1.2m

= 0.60 kN/m

STUD WORK = 0.5 kN/m^2 x HEIGHT OF STUD PARTITION IN MTS.
(100mm)

= 0.5 kN/m^2 x 1.0m

= 0.50kN/m

THEREFORE TOTAL DEAD LOAD ON LB2= 2.77+ 0.60 + 0.50 = 3.87 kN/m.

LIVE LOAD ON LOFT BEAM, LB2:

PITCHED ROOF = 0.75 kN/m^2 x SPAN OF ROOF IN MTS.

= 0.75 kN/m^2 x 2.2m

= 1.65 kN/m

TIMBER FLOOR = 1.50 kN/m^2 x SPAN OF FLOOR IN MTS.

= 1.5 kN/m^2 x 1.2m

= 1.80 kN /m

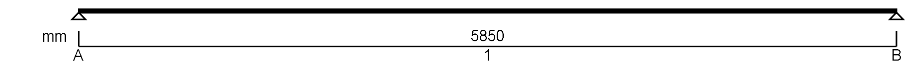
THEREFORE TOTAL LIVE LOAD ON LB2 = 1.65 + 1.80 = 3.45 kN/m

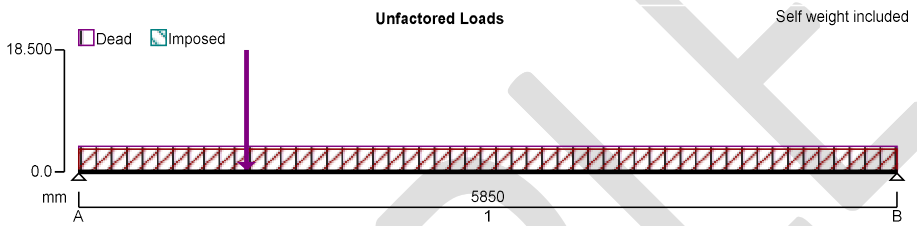
POINT LOAD FROM STAIR TRIMMER 3 = 18.50 kN@1.20m from L.H.S

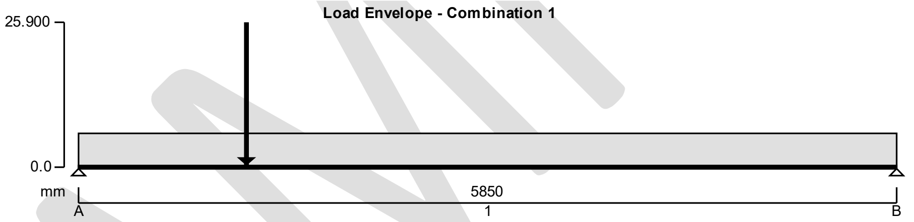
STRUCTURAL REPORT FOR LOFT BEAM (LB2):

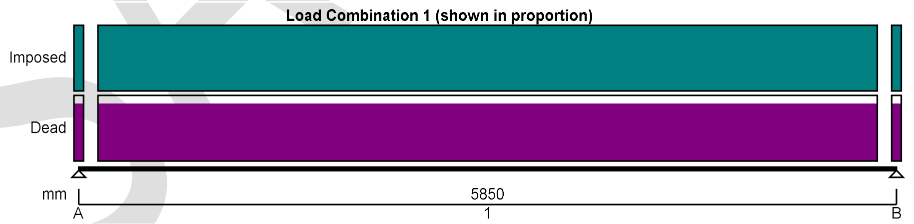
STEEL BEAM ANALYSIS & DESIGN (BS5950)

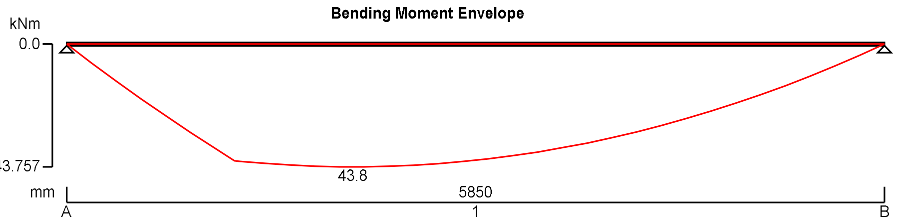
TEDDS calculation version 1.0.05

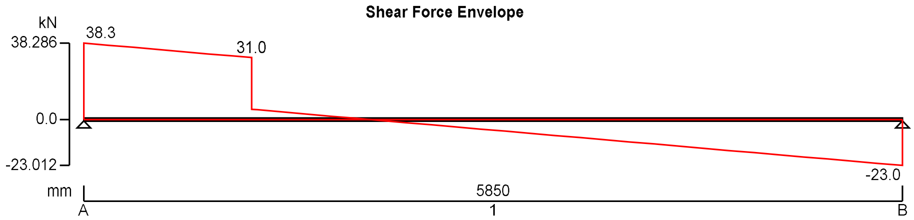












Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 3.87 kN/m
Live full UDL 3.45 kN/m
Dead point load 18.5 kN at 1200 mm

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
	Span 1	Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

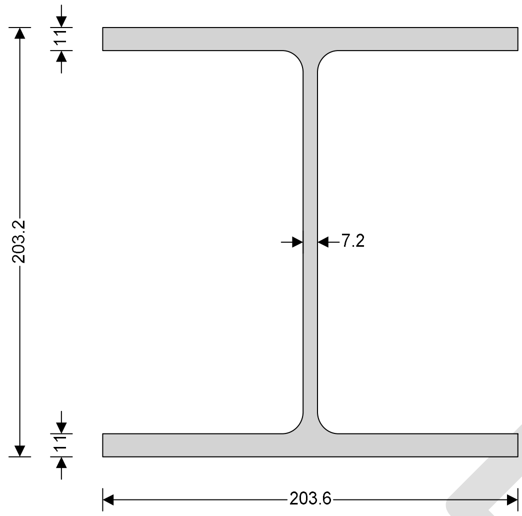
Maximum moment	M_{max} = **43.8** kNm	M_{min} = **0** kNm
Maximum moment span1	M_{s1_max} = **43.8** kNm	M_{s1_min} = **0** kNm
Maximum shear	V_{max} = **38.3** kN	V_{min} = **-23** kN
Maximum shear span1	V_{s1_max} = **38.3** kN	V_{s1_min} = **-23** kN
Deflection span1	δ_{s1_max} = **11.9** mm	δ_{s1_min} = **0** mm
Reactions at support A	R_{A_max} = **38.3** kN	R_{A_min} = **38.3** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **27.3** kN	
Reactions at support B	R_{B_max} = **23** kN	R_{B_min} = **23** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **16.4** kN	

Section details

Section type	**UC 203x203x46**
Steel grade	**S355**

From table 9: Design strength p_y

Thickness of element	max(T, t) = **11.0** mm
Design strength	p_y = **355** N/mm^2
Modulus of elasticity	E = **205000** N/mm^2



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis K_x = **1.00**

Effective length factor in minor axis K_y = **1.00**

Effective length factor for lateral-torsional buckling $K_{LT.A}$ = **1.00**

$K_{LT.B}$ = **1.00**

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **0.88**

Internal compression parts - Table 11

Depth of section d = **160.8** mm

$d / t = 25.4 \times \varepsilon <= 80 \times \varepsilon$ Class 1 plastic

Outstand flanges - Table 11

Width of section b = B / 2 = **101.8** mm

$b / T = 10.5 \times \varepsilon <= 15 \times \varepsilon$ Class 3 semi-compact

Section is class 3 semi-compact

Shear capacity - Section 4.2.3

Design shear force $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **38.3** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area $A_v = t \times D$ = **1463** mm^2

Design shear resistance $P_v = 0.6 \times p_y \times A_v$ = **311.6** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **43.8** kNm

Effective plastic modulus - Section 3.5.6

Limiting value for class 2 compact flange β_{2f} = **8.801**

Limiting value for class 3 semi-compact flange β_{3f} = **13.202**

Limiting value for class 2 compact web β_{2w} = **88.014**

Limiting value for class 3 semi-compact web β_{3w} = **105.617**

Effective plastic modulus - cl.3.5.6.2

$S_{eff} = \min(Z_{xx} + (S_{xx} - Z_{xx}) \times \min([((\beta_{3w} / (d / t))^2 - 1) / ((\beta_{3w} / \beta_{2w})^2 - 1)], [(\beta_{3f} / (b / T) - 1) / (\beta_{3f} / \beta_{2f} - 1)]), S_{xx})$ = **490411** mm^3

Moment capacity - Section 4.2.5

Moment capacity low shear - cl.4.2.5.2	$M_c = \min(p_y \times S_{eff}, 1.2 \times p_y \times Z_{xx})$ = **174.1** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling	$L_E = 1.0 \times L_{s1}$ = **5850** mm
Slenderness ratio	$\lambda = L_E / r_{yy}$ = **113.940**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter	u = **0.847**
Torsional index	x = **17.713**
Slenderness factor	$v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25}$ = **0.756**
Ratio - cl.4.3.6.9	$\beta_W = S_{eff} / S_{xx}$ = **0.986**
Equivalent slenderness - cl.4.3.6.7	$\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]}$ = **72.355**
Limiting slenderness - Annex B.2.2	$\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5}$ = **30.198**

$\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant	α_{LT} = **7.0**
Perry factor	$\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0)$ = **0.295**
Euler stress	$p_E = \pi^2 \times E / \lambda_{LT}^2$ = **386.5** N/mm^2
	$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2$ = **427.8** N/mm^2
Bending strength - Annex B.2.1	$p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5})$ = **213.8** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment	M_2 = **42.7** kNm
Moment at centre-line of segment	M_3 = **41.4** kNm
Moment at three quarter point of segment	M_4 = **27.2** kNm
Maximum moment in segment	M_{abs} = **43.8** kNm

Equivalent uniform moment factor for lateral-torsional buckling

$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44)$ = **0.913**

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment	$M_b = p_b \times S_{eff}$ = **104.8** kNm
	M_b / m_{LT} = **114.8** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead, imposed, live and other loads

Limiting deflection	$\delta_{lim} = L_{s1} / 360$ = **16.3** mm
Maximum deflection span 1	$\delta = \max(abs(\delta_{max}), abs(\delta_{min}))$ = **11.861** mm

PASS - Maximum deflection does not exceed deflection limit

LOAD CALCULATIONS FOR LOFT BEAM (LB3):

CLEAR SPAN OF THE LOFT BEAM (LB1) = 5850mm

LOADING:

DEAD LOAD ON LOFT BEAM (LB3):

TIMBER FLOOR = 0.50 kN/m^2 x SPAN OF FLOOR IN MTS.

= 0.50 kN/m^2 x 2.00m

= 1.00 kN/m

STUD WORK = 0.5kN /m2 x HEIGHT OF STUD PARTITION IN MTS.
(100mm)

= 0.5 kN/m^2 x 2.3m

= 1.15kN/m

THEREFORE TOTAL DEAD LOAD ON LB3 = 1.00 + 1.15 = 2.15 kN/m.

LIVE LOAD ON LOFT BEAM, LB3:

TIMBER FLOOR = 1.50 kN/m^2 x SPAN OF FLOOR IN MTS.

= 1.5 kN/m^2 x 2.00m

= 3.00 kN /m

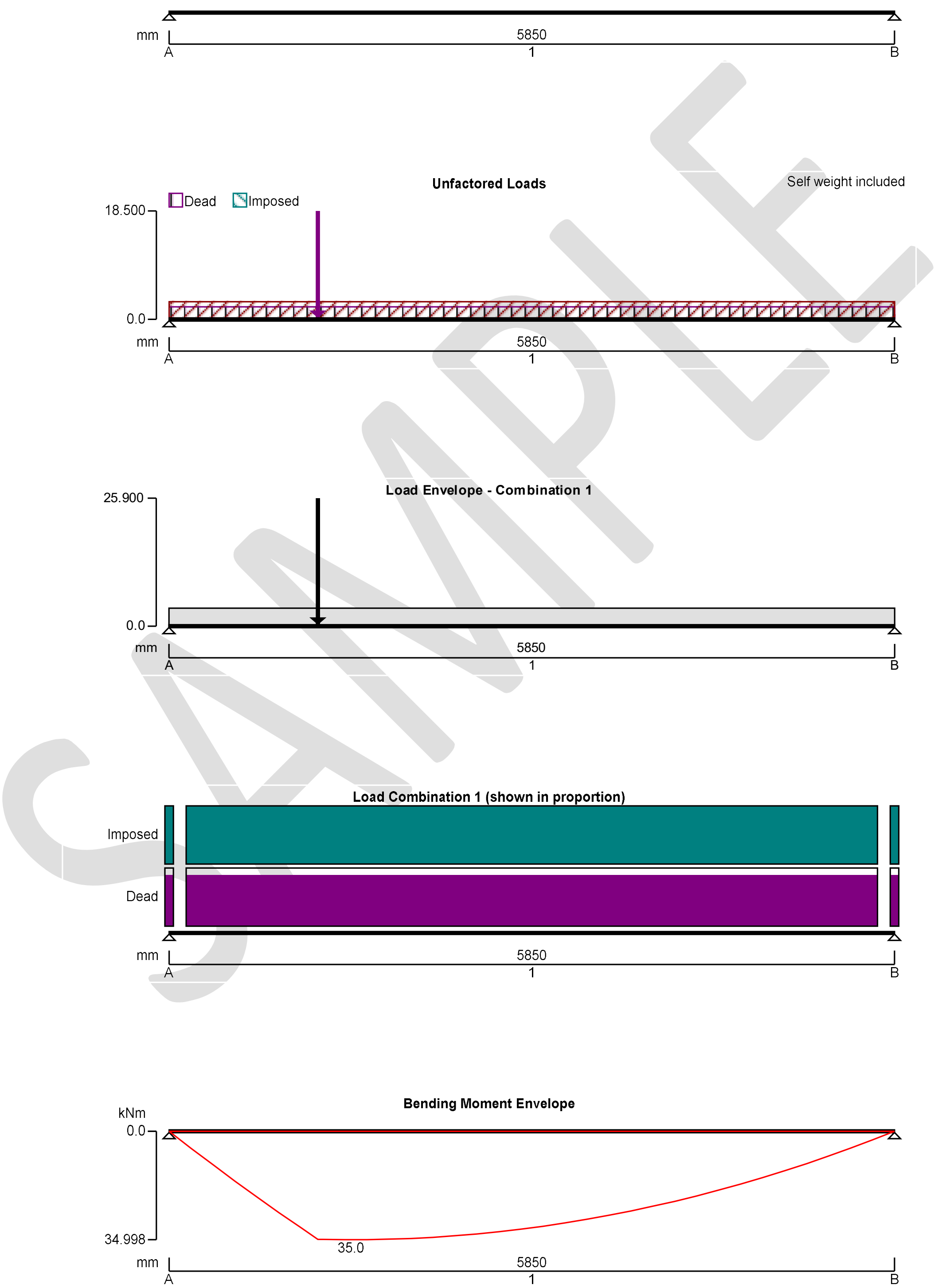
THEREFORE TOTAL LIVE LOAD ON LB3 = 3.00 kN/m

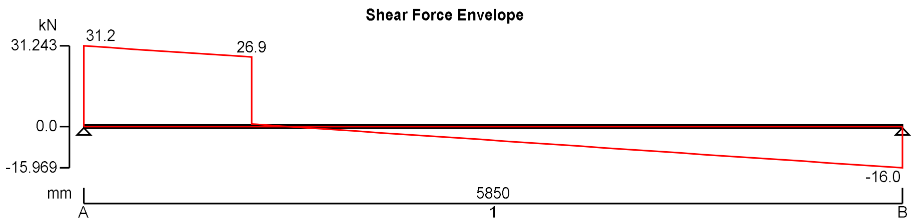
POINT LOAD FROM STAIR TRIMMER 3 = 18.50 kN@1.20m from L.H.S

STRUCTURAL REPORT FOR LOFT BEAM (LB3):

STEEL BEAM ANALYSIS & DESIGN (BS5950)

TEDDS calculation version 1.0.05





Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 2.15 kN/m
Live full UDL 3 kN/m
Dead point load 18.5 kN at 1200 mm

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
	Span 1	Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

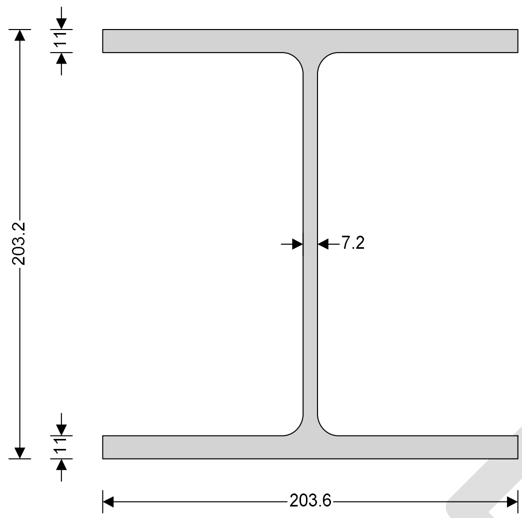
Maximum moment	M_{max} = **35** kNm	M_{min} = **0** kNm
Maximum moment span1	M_{s1_max} = **35** kNm	M_{s1_min} = **0** kNm
Maximum shear	V_{max} = **31.2** kN	V_{min} = **-16** kN
Maximum shear span1	V_{s1_max} = **31.2** kN	V_{s1_min} = **-16** kN
Deflection span1	δ_{s1_max} = **9.1** mm	δ_{s1_min} = **0** mm
Reactions at support A	R_{A_max} = **31.2** kN	R_{A_min} = **31.2** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **22.3** kN	
Reactions at support B	R_{B_max} = **16** kN	R_{B_min} = **16** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **11.4** kN	

Section details

Section type	**UC 203x203x46**
Steel grade	**S355**

From table 9: Design strength p_y

Thickness of element	max(T, t) = **11.0** mm
Design strength	p_y = **355** N/mm^2
Modulus of elasticity	E = **205000** N/mm^2



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis — K_x = **1.00**

Effective length factor in minor axis — K_y = **1.00**

Effective length factor for lateral-torsional buckling — $K_{LT.A}$ = **1.00**

$K_{LT.B}$ = **1.00**

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **0.88**

Internal compression parts - Table 11

Depth of section — d = **160.8** mm

$d / t = 25.4 \times \varepsilon <= 80 \times \varepsilon$ — Class 1 plastic

Outstand flanges - Table 11

Width of section — b = B / 2 = **101.8** mm

$b / T = 10.5 \times \varepsilon <= 15 \times \varepsilon$ — Class 3 semi-compact

Section is class 3 semi-compact

Shear capacity - Section 4.2.3

Design shear force — $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **31.2** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area — $A_v = t \times D$ = **1463** mm^2

Design shear resistance — $P_v = 0.6 \times p_y \times A_v$ = **311.6** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment — $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **35** kNm

Effective plastic modulus - Section 3.5.6

Limiting value for class 2 compact flange — β_{2f} = **8.801**

Limiting value for class 3 semi-compact flange — β_{3f} = **13.202**

Limiting value for class 2 compact web — β_{2w} = **88.014**

Limiting value for class 3 semi-compact web — β_{3w} = **105.617**

Effective plastic modulus - cl.3.5.6.2

$S_{eff} = \min(Z_{xx} + (S_{xx} - Z_{xx}) \times \min([((\beta_{3w} / (d / t))^2 - 1) / ((\beta_{3w} / \beta_{2w})^2 - 1)], [(\beta_{3f} / (b / T) - 1) / (\beta_{3f} / \beta_{2f} - 1)]), S_{xx})$ = **490411** mm^3

Moment capacity - Section 4.2.5

Moment capacity low shear - cl.4.2.5.2 $M_c = \min(p_y \times S_{eff}, 1.2 \times p_y \times Z_{xx})$ = **174.1** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling $L_E = 1.0 \times L_{s1}$ = **5850** mm

Slenderness ratio $\lambda = L_E / r_{yy}$ = **113.940**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter u = **0.847**

Torsional index x = **17.713**

Slenderness factor $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25}$ = **0.756**

Ratio - cl.4.3.6.9 $\beta_W = S_{eff} / S_{xx}$ = **0.986**

Equivalent slenderness - cl.4.3.6.7 $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]}$ = **72.355**

Limiting slenderness - Annex B.2.2 $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5}$ = **30.198**

$\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant α_{LT} = **7.0**

Perry factor $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0)$ = **0.295**

Euler stress $p_E = \pi^2 \times E / \lambda_{LT}^2$ = **386.5** N/mm²

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2$ = **427.8** N/mm²

Bending strength - Annex B.2.1 $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5})$ = **213.8** N/mm²

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment M_2 = **35** kNm

Moment at centre-line of segment M_3 = **31.1** kNm

Moment at three quarter point of segment M_4 = **19.5** kNm

Maximum moment in segment M_{abs} = **35** kNm

Equivalent uniform moment factor for lateral-torsional buckling

$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44)$ = **0.878**

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment $M_b = p_b \times S_{eff}$ = **104.8** kNm

M_b / m_{LT} = **119.4** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead, imposed, live and other loads

Limiting deflection $\delta_{lim} = L_{s1} / 360$ = **16.3** mm

Maximum deflection span 1 $\delta = \max(abs(\delta_{max}), abs(\delta_{min}))$ = **9.072** mm

PASS - Maximum deflection does not exceed deflection limit

LOAD CALCULATIONS FOR TIMBER FLOOR JOISTS:

SPAN OF FLOOR JOISTS = 2450mm

ASSUME FLOOR JOISTS = 50mm x 150mm

TIMBER GRADE = C24

JOISTS CENTERS = 400mm

LOADING:

DEAD LOAD ON TIMBER FLOOR JOISTS:

TIMBER FLOOR = 0.50 kN/m^2 x SPAN OF FLOOR IN MTS.

= 0.50 kN/m^2 x 0.4m

= 0.20 kN/m

THEREFORE TOTAL DEAD LOAD ON TIMBER FLOOR JOISTS= 0.20kN/m

LIVE LOAD ON TIMBER FLOOR JOISTS

TIMBER FLOOR = 1.5 kN/m^2 x SPAN OF FLOOR IN MTS.

= 1.5 kN/m^2 x 0.4m

= 0.60 kN/m

THEREFORE TOTAL LIVE LOAD ON TIMBER FLOOR JOISTS = 0.60 kN/m

LOAD CALCULATIONS FOR TIMBER FLOOR JOISTS:

Analysis for a simply-supported single-span timber beam to BS 5268

TEDDS calculation version 1.0.02

Span length & partial factors for loading

Span (mm)	Factors for moments & forces			Factors for deflection		
	γ_{fd}	γ_{fi}	γ_{fw}	γ_{dd}	γ_{di}	γ_{dw}
2450	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**

Loading data (unfactored)

Ref.	Category	Type	Load kN/m	Position mm	Load kN/m	Position mm
1	**"Dead"**	UDL	**0.2**	**0**	-	**2450**
2	**"Imposed"**	UDL	**0.6**	**0**	-	**2450**

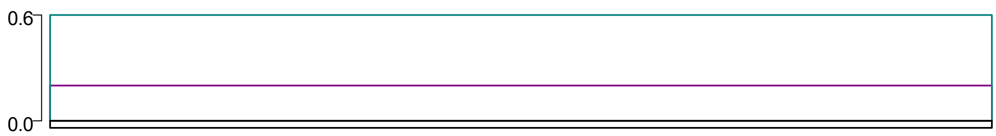
Analysis results - entire span

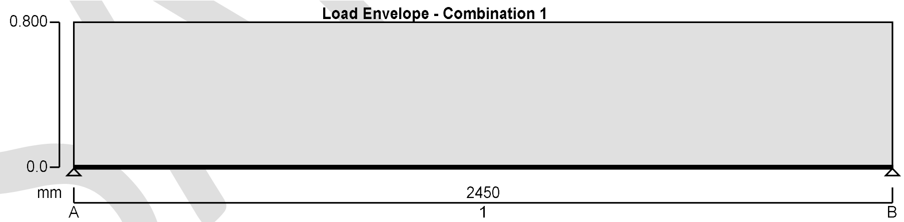
R_a	R_b	V	M		Deflection: δEI	
kN (fac)	kN (fac)	kN (fac)	kNm (fac)	Sense	kNm^3	Direction
1.0	**1.0**	**1.0**	**0.6**	**"Sagging"**	**0.38**	**"Down"**

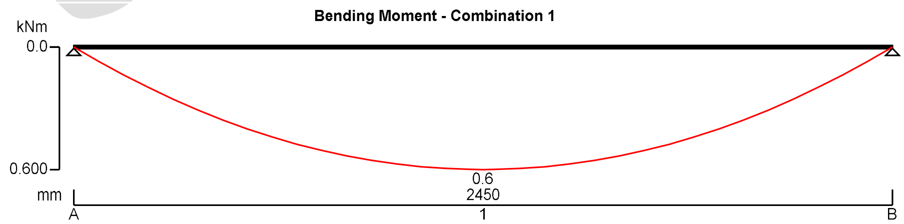
Unfactored support reactions

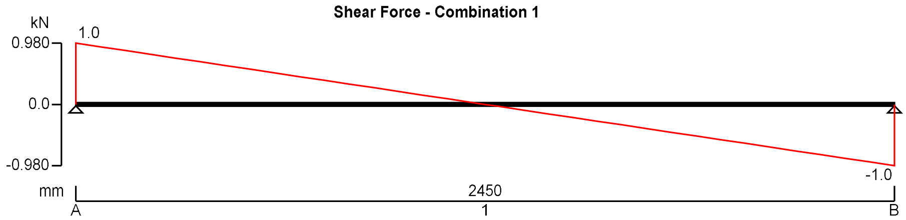
Support A	Dead load **-0.2** kN	Live load **-0.7** kN	Wind load **0.0** kN
Support B	Dead load **-0.2** kN	Live load **-0.7** kN	Wind load **0.0** kN

Beam Loads









Member design checks for a simply-supported single-span timber beam to BS 5268

<u>Timber member design BS 5268-2:2002</u>

Summary of results				
Section size	D = **200** mm	B = **50** mm	A= **10000** mm^2	
Section properties (x-x)	I_{xx} = **33333333** mm^4	Z_{xx} = **333333** mm^3	r_{xx} = **57.7** mm	
(y-y)	I_{yy} = **2083333** mm^4	Z_{yy} = **83333** mm^3	r_{yy} = **14.4** mm	
Grade	**"C24"**	σ_c = **7.90** N/ mm^2		
Check	*Stress*	*Capacity*	*Notes*	*Result*
Bending stress	$\sigma_{m.a.para} = M / Z_{xx}$ = **1.80** N/mm^2	$\sigma_{m.adm.para}$ = **7.84** N/mm^2	Moment M = **0.6** kNm	**Pass**
Shear stress	τ_a = **0.15** N/mm^2	τ_{adm} = **0.71** N/mm^2	Shear V = **1.0** kN	**Pass**
Deflection	δ = **1.1** mm	14 mm		**Pass**
	δ / L_s = **0.00047**	0.003		**Pass**

LOAD CALCULATIONS FOR FLAT ROOF JOISTS:

SPAN OF JOISTS = 3650mm

ASSUME FLAT ROOF = 50mm x 170mm

TIMBER GRADE = C24

JOISTS CENTERS = 400mm

LOADING:

DEAD LOAD ON FLAT ROOF JOSITS:

FLAT ROOF = 0.81 kN/m^2 x SPAN OF ROOF IN MTS.

= 0.81 kN/m^2 x 0.4m

= 0.32kN/m

THEREFORE TOTAL DEAD LOAD ON FLAT ROOF JOISTS= 0.32 kN/m

LIVE LOAD ON FLAT ROOF JOISTS:

PITCHED ROOF = 0.75 kN / m2 x SPAN OF ROOF IN MTS.

= 0.75 kN/m^2 x 0.4m

= 0.30 kN/m

THEREFORE TOTAL LIVE LOAD ON FLAT ROOF JOISTS = 0.30 kN/m

LOAD CALCULATIONS FOR FLAT ROOF JOISTS:

Analysis for a simply-supported single-span timber beam to BS 5268

TEDDS calculation version 1.0.02

Span length & partial factors for loading

Span (mm)	Factors for moments & forces			Factors for deflection		
	γ_{fd}	γ_{fi}	γ_{fw}	γ_{dd}	γ_{di}	γ_{dw}
3650	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**

Loading data (unfactored)

Ref.	Category	Type	Load kN/m	Position mm	Load kN/m	Position mm
1	**"Dead"**	UDL	**0.3**	**0**	-	**3650**
2	**"Imposed"**	UDL	**0.3**	**0**	-	**3650**

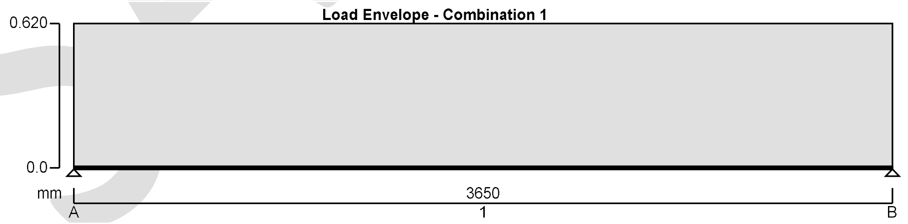
Analysis results - entire span

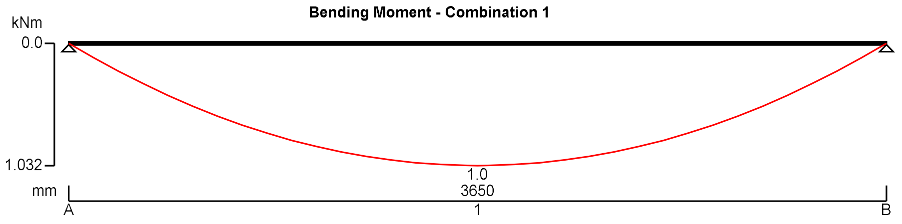
R_a	R_b	V	M		Deflection: δEI	
kN (fac)	kN (fac)	kN (fac)	kNm (fac)	Sense	kNm^3	Direction
1.1	**1.1**	**1.1**	**1.0**	**"Sagging"**	**1.43**	**"Down"**

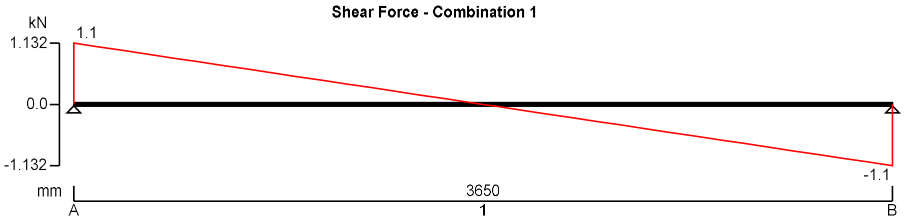
Unfactored support reactions

Support A	Dead load **-0.6** kN	Live load **-0.5** kN	Wind load **0.0** kN
Support B	Dead load **-0.6** kN	Live load **-0.5** kN	Wind load **0.0** kN

Beam Loads







Member design checks for a simply-supported single-span timber beam to BS 5268

<u>Timber member design BS 5268-2:2002</u>

Summary of results				
Section size	D = **170** mm	B = **50** mm	A= **8500** mm^2	
Section properties (x-x)	I_{xx} = **20470833** mm^4	Z_{xx} = **240833** mm^3	r_{xx} = **49.1** mm	
(y-y)	I_{yy} = **1770833** mm^4	Z_{yy} = **70833** mm^3	r_{yy} = **14.4** mm	
Grade	**"C24"**	σ_c = **7.90** N/ mm^2		
Check	*Stress*	*Capacity*	*Notes*	*Result*
Bending stress	$\sigma_{m.a.para}$ = M / Z_{xx} = **4.29** N/mm^2	$\sigma_{m.adm.para}$ = **7.98** N/mm^2	Moment M = **1.0** kNm	**Pass**
Shear stress	τ_a = **0.20** N/mm^2	τ_{adm} = **0.71** N/mm^2	Shear V = **1.1** kN	**Pass**
Deflection	δ = **6.7** mm	14 mm		**Pass**
	δ / L_s = **0.00183**	0.003		**Pass**

LOAD CALCULATIONS FOR TIMBER RAFTERS:

SPAN OF RAFTERS = 3000mm

ASSUME RAFTERS = 50mm x 150mm

SLOPE = 30-45 DEGREES

TIMBER GRADE = C24

JOISTS CENTERS = 400mm

LOADING:

DEAD LOAD ON TIMBER RAFTERS:

PITCHED ROOF = 1.26 kN/m^2 x SPAN OF ROOF IN MTS.

= 1.26 kN/m^2 x 0.4m

= 0.50 kN/m

THEREFORE TOTAL DEAD LOAD ON TIMBER RAFTERS = 0.50 kN/m

LIVE LOAD ON TIMBER RAFTERS:

PITCHED ROOF = 0.75 kN/m^2 x SPAN OF ROOF IN MTS.

= 0.75 kN/m^2 x 0.4m

= 0.30 kN/m

THEREFORE TOTAL LIVE LOAD ON TIMBER RAFTERS = 0.30 kN/m

LOAD CALCULATIONS FOR TIMBER RAFTERS:

Analysis for a simply-supported single-span timber beam to BS 5268

TEDDS calculation version 1.0.02

Span length & partial factors for loading

Span	Factors for moments & forces			Factors for deflection		
(mm)	γ_{fd}	γ_{fi}	γ_{fw}	γ_{dd}	γ_{di}	γ_{dw}
3000	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**

Loading data (unfactored)

Ref.	Category	Type	Load kN/m	Position mm	Load kN/m	Position mm
1	**"Dead"**	UDL	**0.5**	**0**	-	**3000**
2	**"Imposed"**	UDL	**0.3**	**0**	-	**3000**

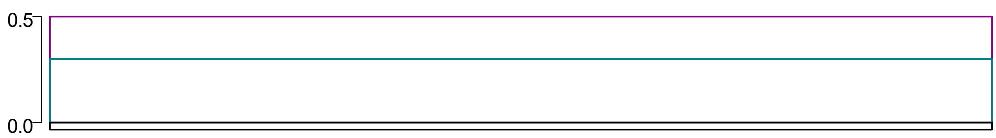
Analysis results - entire span

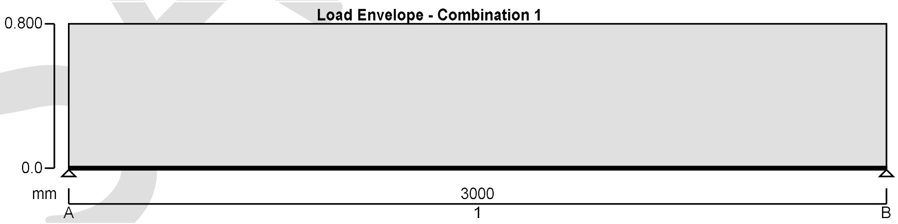
R_a	R_b	V	M		Deflection: δEI	
kN (fac)	kN (fac)	kN (fac)	kNm (fac)	Sense	kNm^3	Direction
1.2	**1.2**	**1.2**	**0.9**	**"Sagging"**	**0.84**	**"Down"**

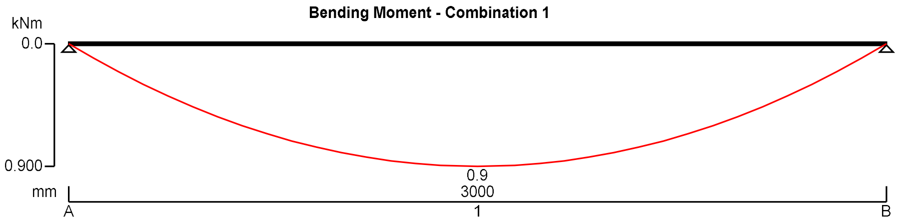
Unfactored support reactions

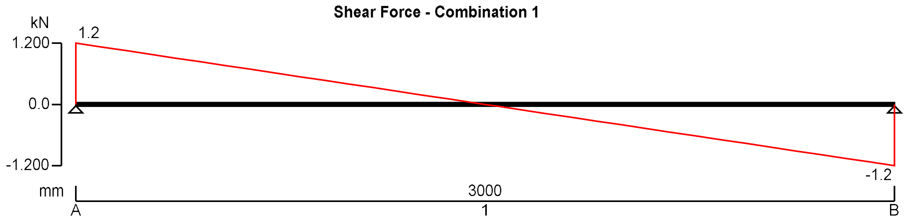
Support A	Dead load **-0.7** kN	Live load **-0.4** kN	Wind load **0.0** kN
Support B	Dead load **-0.7** kN	Live load **-0.4** kN	Wind load **0.0** kN

Beam Loads









Member design checks for a simply-supported single-span timber beam to BS 5268

Timber member design BS 5268-2:2002

Summary of results				
Section size	D = **150** mm	B = **50** mm	A= **7500** mm^2	
Section properties (x-x)	I_{xx} = **14062500** mm^4	Z_{xx} = **187500** mm^3	r_{xx} = **43.3** mm	
(y-y)	I_{yy} = **1562500** mm^4	Z_{yy} = **62500** mm^3	r_{yy} = **14.4** mm	
Grade	**"C24"**	σ_c = **7.90** N/ mm^2		
Check	*Stress*	*Capacity*	*Notes*	*Result*
Bending stress	$\sigma_{m.a.para}$ = M / Z_{xx} = **4.80** N/mm^2	$\sigma_{m.adm.para}$ = **8.09** N/mm^2	Moment M = **0.9** kNm	**Pass**
Shear stress	τ_a = **0.24** N/mm^2	τ_{adm} = **0.71** N/mm^2	Shear V = **1.2** kN	**Pass**
Deflection	δ = **5.8** mm	14 mm		**Pass**
	δ / L_s = **0.00192**	0.003		**Pass**

DESIGN OF MILD STEEL PLATE SPREADER:

1. SPREADER DESIGN FOR LOFT BEAM, LB1 & LB2 [WORST CASE]:

Maximum reaction from loft beam = 40.00 kN x 0.7 = 28.00kN [Unfactored]

As per Structural Renovation of Traditional Buildings (CIRIA-REPORT:1986)

The basic compressive stress = 0.42 N/mm^2 and Local bearing factor is 1.5

So the allowable compressive stress = 0.42 N/mm^2 x 1.5 = 0.63N/mm^2

Length of Spreader is (28.00 x 1000) / (0.63 x 100) = 445mm

Therefore provide 500mm Long x 100mm Wide x 25mm THICK Mild Steel Plate As Spreader

2. SPREADER DESIGN FOR ROOF BEAM (RB) [WORST CASE]:

Maximum reaction from roof beam = 25.00 kN x 0.7 = 17.50 kN [Unfactored]

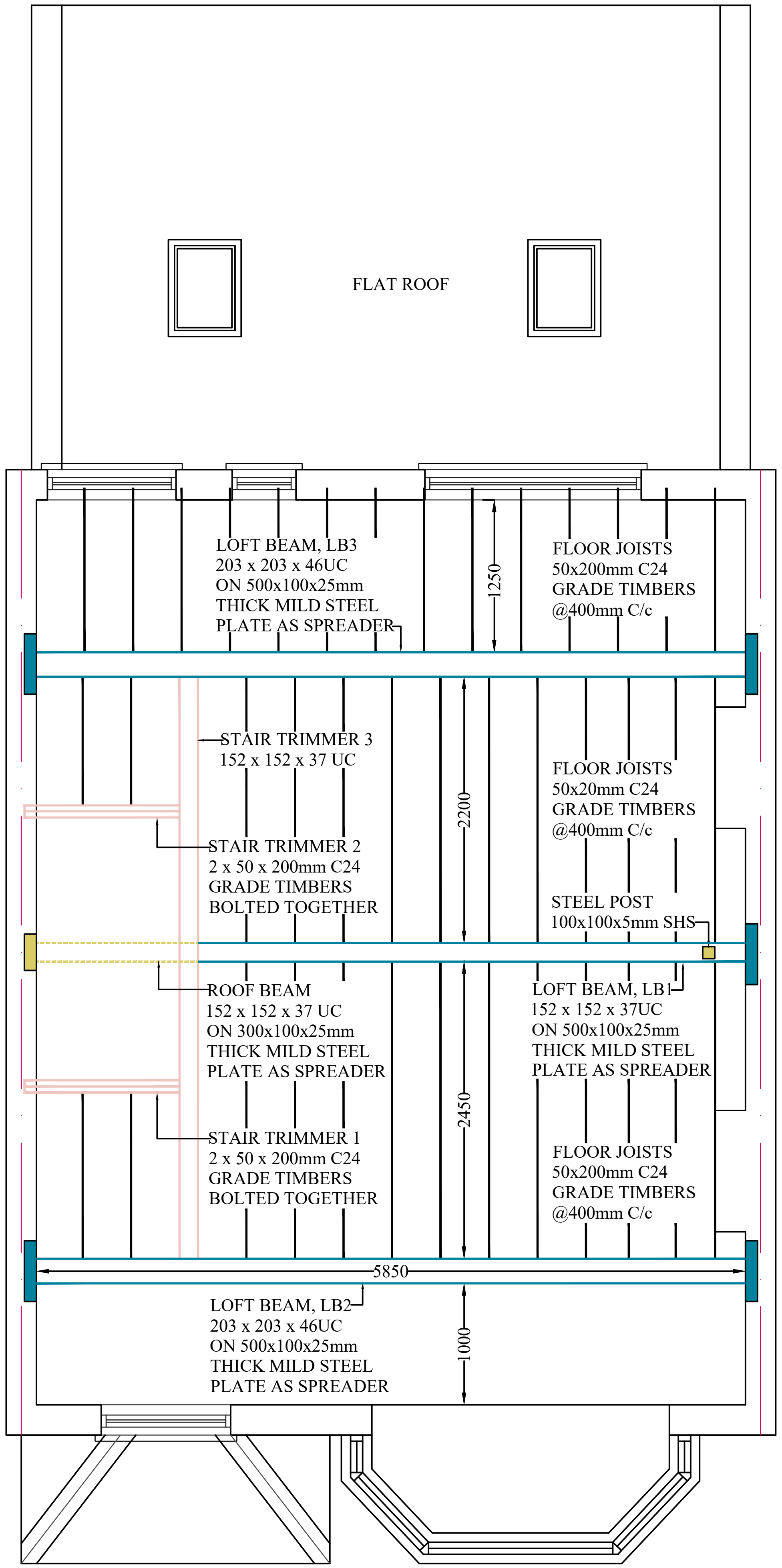
As per Structural Renovation of Traditional Buildings (CIRIA-REPORT:1986)

The basic compressive stress = 0.42 N/mm^2 and Local bearing factor is 1.5

So the allowable compressive stress = 0.42 N/mm^2 x 1.5 = 0.63N/mm^2

Length of Spreader is (17.50 x 1000) / (0.63 x 100) = 278mm

Therefore provide 300mm Long x 100mm Wide x 25mm THICK Mild Steel Plate As Spreader



SAMPLE DRAWING

PROJECT	SE19 3HX
DESCRIPTION	PROPOSED LOFT FLOOR STRUCTURAL PLAN
SCALE	NOT TO SCALE
DRAWINGS NO.	S01
DATE	24-12-2025

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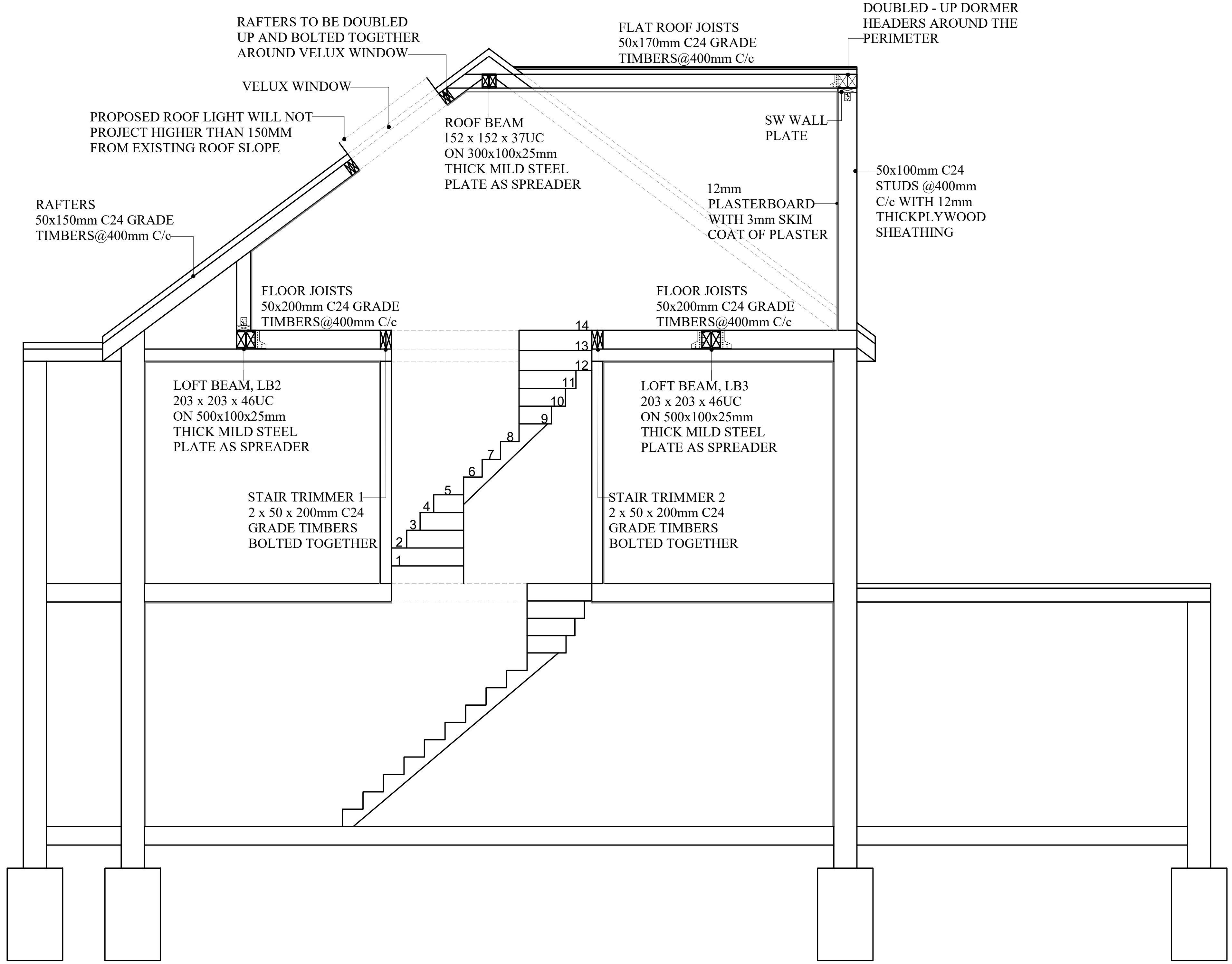

PROJECT	SE19 3HX
DESCRIPTION	PROPOSED SECTION
SCALE	NOT TO SCALE
DRAWINGS NO.	S02
DATE	24-12-2025

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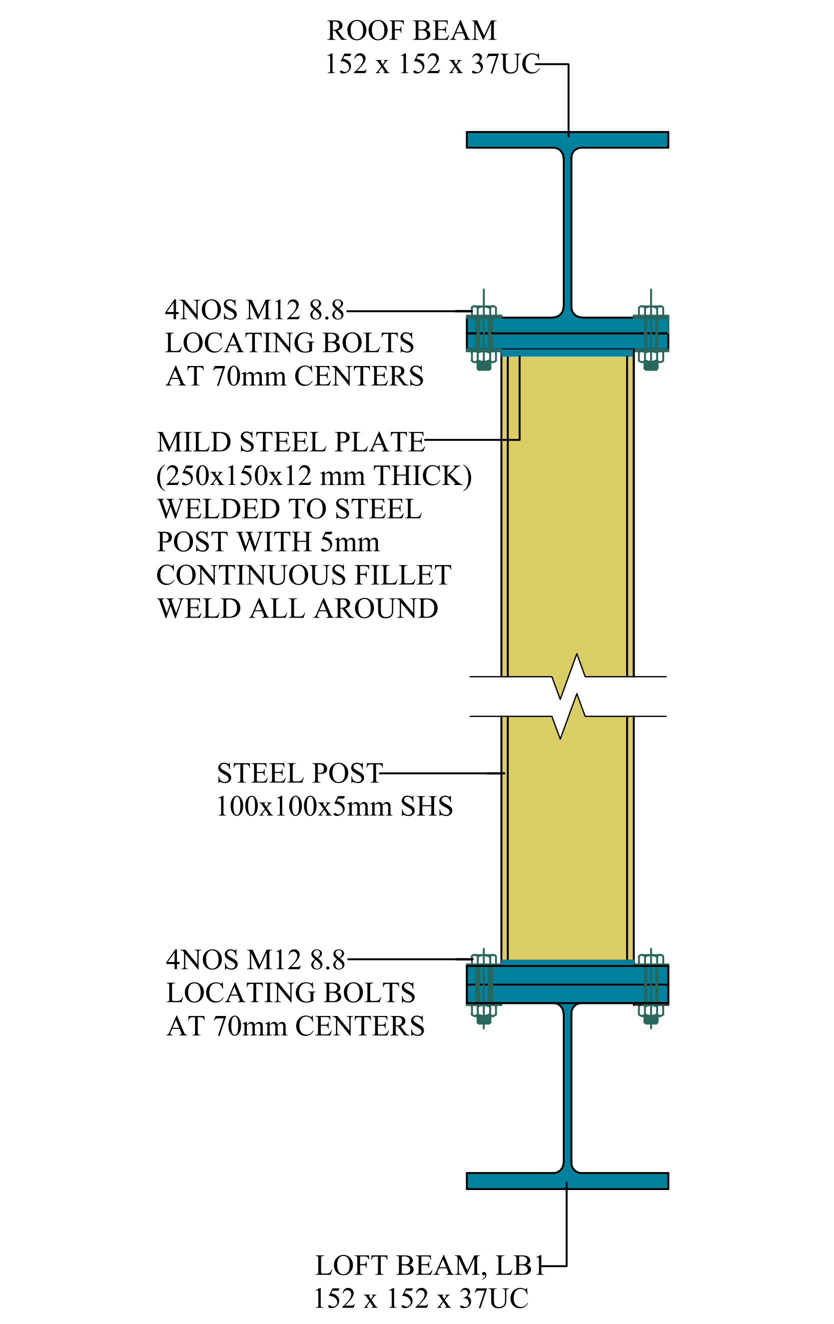
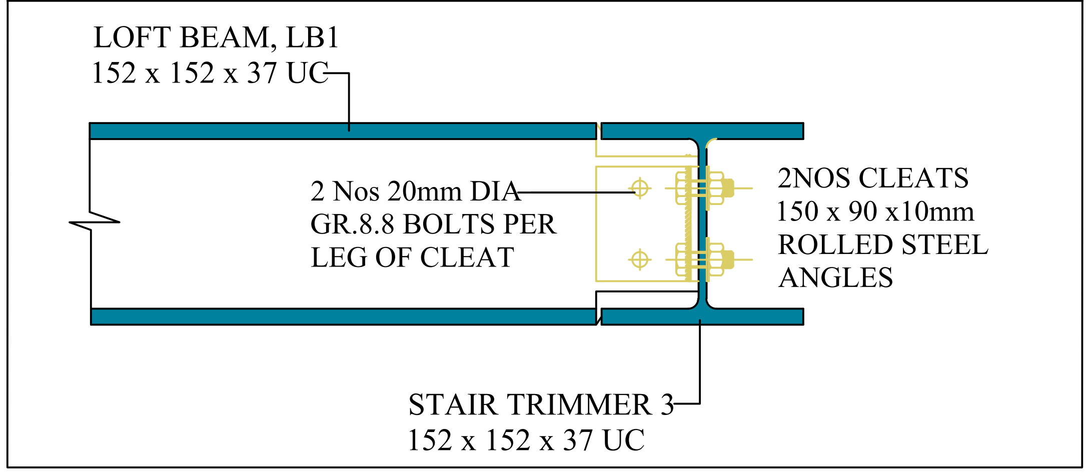
SAMPLE DRAWING

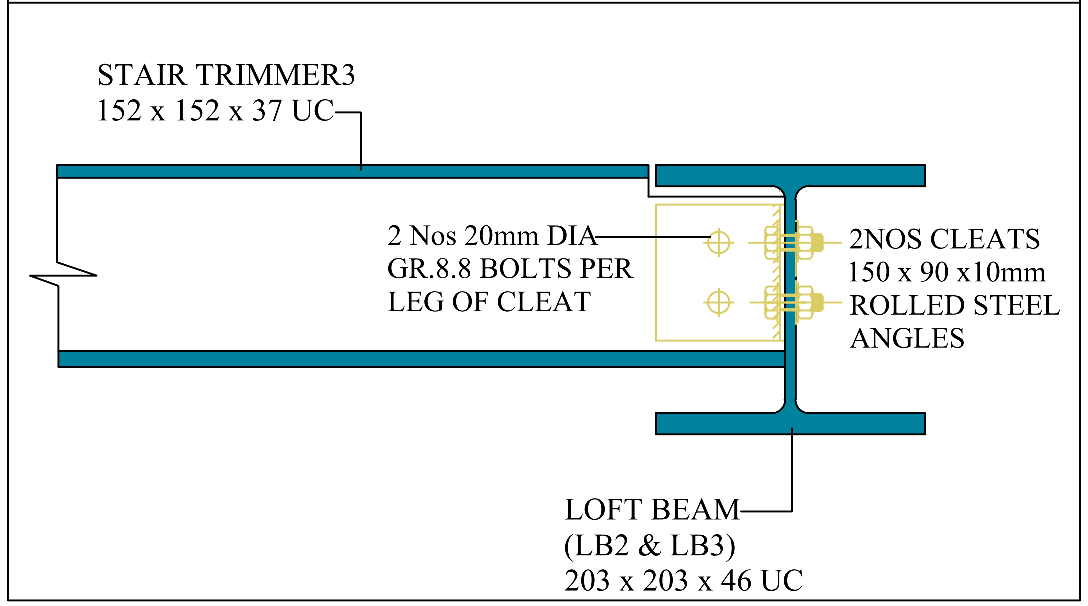
ROOF BEAM TO STEEL POST TO LOFT BEAM CONNECTION



LOFT BEAM, LB1 TO STAIR TRIMMER 3 CONNECTION DETAILS



STAIR TRIMMER 3 TO LOFT BEAM, LB2 & LB3 CONNECTION DETAILS



PROJECT	SE19 3HX
DESCRIPTION	CONNECTIONS
SCALE	NOT TO SCALE
DRAWINGS NO.	S03
DATE	24-12-2025

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SAMPLE DRAWING